

# EXPERIMENTAL TESTING AND DESIGN OF BRB WITH BOLTED AND PINNED CONNECTIONS

Stefan Wijanto<sup>1</sup> and G. Charles Clifton<sup>2</sup>

## SUMMARY

The recent series of damaging earthquakes in Christchurch, New Zealand has encouraged greater recognition of the post-earthquake economic impacts on New Zealand society and higher emphasis on low-damage earthquake resisting systems. Braced frames incorporating Buckling Restrained Braces (BRB) are seen as a significant contender for such a system.

This research project focuses on the development of a reliable design procedure and detailing requirements for a generic BRB system. To gauge the performance of the designed system and to ascertain the reliability of the developed procedure, a series of static and dynamic sub-assembly tests on the BRB frame with two different brace connection configurations were performed. The results are presented and discussed herein.

The experimental tests generated stable and near symmetrical hysteresis loops, which is a principal characteristic of a well performing BRB system, albeit with the occurrence of slack in the connections. The experimental test results shows that several improvements need to be made to the proposed design procedure and detailing as outlined throughout the paper; especially the procedural modification to prevent slack from occurring in the two different connection systems. It is envisaged that applications will typically involve use of proprietary braces, however these need to be applied in accordance with the New Zealand design procedure; and determining the appropriate procedure was a key part of this project.

## INTRODUCTION

New Zealand is located in a moderate to high seismic region. Recently, Christchurch and its surrounding regions have been devastated by a series of earthquakes starting from September 4, 2010, with the most intense earthquake recorded on February 22, 2011, generating Modified Mercalli scale of 8 to 10 within the Central Business District (CBD) and followed by over ten thousand recorded aftershocks. This earthquake series caused significant damage and failure of unreinforced masonry buildings and the February 22 2011 earthquake caused structural damage to all buildings in the CBD. Despite this, steel structures located in Christchurch have performed beyond expectations considering the severity of the earthquake, most notably structures with a seismic resisting system such as Eccentrically Braced Frames (EBFs) or Moment Resisting Frames (MRFs) [1]. However, repair of braced structures which are designed and built according to conventional procedures costs time and impacts on the economy of the recovering city.

With the increased recognition of the post-earthquake economic impacts on society has come the increased demand for seismic resisting systems that will deliver a high damage threshold in severe earthquakes, allowing buildings to be rapidly returned to service and requiring little or no structural repair. This civil engineering research focuses on the development of one such system.

The Buckling Restrained Brace (BRB) system (see Figure 1 for the anatomy of a brace) has been developed and applied in

both North America and Japan, being recognised for its superior seismic performance compared to existing braced systems due to its suppression of brace compression buckling. Because of this, a BRB has similar strength and stiffness in tension and compression which simplifies design of the seismic-resisting system containing the braces. However, in these countries, they have been implemented into the market as proprietary items for use in conjunction with the relevant nationally based design procedure. This does not suit their application in New Zealand. The concept of the BRB system itself is not proprietary, but the configuration and details of each specific brace assembly is commonly subjected to US patent laws [2]. Its first use in New Zealand was in 2000 in the seismic upgrade of the Geology Department Building at the University of Canterbury; design by Holmes Consulting Group, Christchurch. On a visual inspection by the second author following the 22 February 2011 event it had performed well, although the intensity of shaking at the University was considerably lower than in the CBD.

Furthermore, the BRB systems are more sustainable compared to conventional compression/tension concentrically braced frames as they achieve relatively greater strength with less material in comparison, allowing construction with less overall steel. The increased compression capacity of the BRB system lowers and simplifies the foundation and connection design requirements. The system could also simplify the design and detailing requirements for the collector beams and the columns.

<sup>1</sup> Structural Engineer, Holmes Consulting Group, Hamilton, New Zealand

<sup>2</sup> Associate Professor, Department of Civil and Environmental Engineering, University of Auckland, Auckland, New Zealand

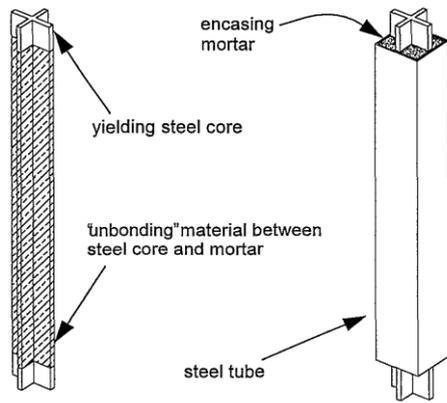


Figure 1: Anatomy of BRB System [3].

To allow widespread use of the BRB system in New Zealand, a reliable generic design method must be developed, its performance verified by experimental testing and the design procedure for its use established in compliance to New Zealand standards and Centrically Braced Frame (CBF) design procedures. To achieve this, a method has been developed and used to design a standard multi-storey building as a case study. An example brace and bay from that case study has then been physically built and experimentally tested on in accordance with the established North American experimental testing regime. These tests performed well, delivering overall the expected performance, and as a result, recommendations have been able to be made for the brace and system designs.

### CONCEPTUAL SUMMARY OF BRB

#### Background

When a building is subjected to an earthquake, a significant amount of kinetic energy is distributed into the structure as strain energy and the level of damage sustained by the building depends in part on the dissipation of this energy. Earthquake resisting systems are designed to dissipate energy within the structure by yielding at controlled locations in a ductile manner. A key requirement is that the seismic-resisting system can sustain these demands over several cycles of inelastic loading without significant loss of strength. A BRB frame system is one such earthquake resisting system.

The BRB system is designed to resist structural frame distortions when subjected to lateral loads. In a severe earthquake, the braces are subjected to repeated cycles of loading which exceeds the elastic limits of the braces. These braces will yield in compression and tension to absorb this loading and to suppress inelastic demand in the rest of the structural system. When in compression, a conventional brace cannot yield due to its tendency to buckle due to the combination of compression force and the long distance between points of effective restraint, which results in unsymmetrical hysteretic behaviour as shown in Figure 2. After buckling occurs, the ability of the braced members to resist earthquake loading and to dissipate energy will be severely reduced. This also leads to complex design methods as the behaviour of a buckled brace is degrading and dependent on the brace slenderness. Therefore, this research has been focused towards finding a method to prevent the buckling of the brace, and to ensure that the same strength can be achieved by the brace in both tension and compression, which simplifies the design process.

The BRB system is designed to suppress member and section compression buckling of the brace by providing continuous

lateral restraint (Figure 3), which ensures that the brace length of the core elements is effectively zero. Therefore, the BRB system develops a balanced hysteretic loop when subjected to cyclic loading, with its compression yielding response similar to its tensile response [4]. This behaviour is the distinguishing feature of the BRB system as compared to other conventional braces in terms of hysteretic response shown in Figure 2.

The ability of the BRB system to restrain buckling and its symmetrical hysteretic response gives the system a clear advantage compared to conventional CBF. The full, stable hysteretic response also means less redistribution of loads and deformations in BRB as compared to conventional braces. The ease the designers have in manipulating the area of steel core allows them to make the capacity of each storey closer to the demand easily and thus mitigates the tendency of damage concentration in weak stories. In addition, as the braces do not buckle laterally, damage on adjacent non-structural elements can be reduced [5].

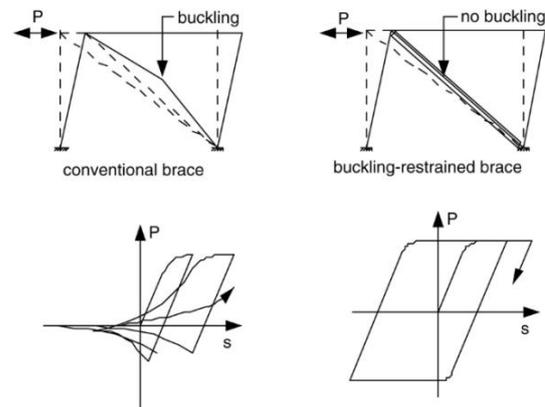


Figure 2: Behaviour of conventional brace and BRB [6].

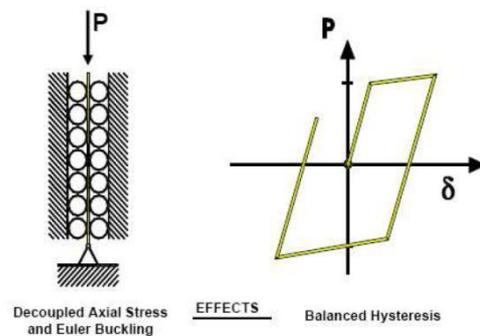


Figure 3: Mechanics of BRB [7].

The behaviour of the BRB system does, however, have several disadvantages. The BRB system has a relatively low post-yield stiffness which may still lead to the concentration of damage in one level, even though the brace capacities can be balanced throughout different storeys. It is a requirement of the secondary elements of the seismic-resisting system to possess sufficient strength and stiffness to prevent this from happening and the design procedure for CBFs, presented in Feeny and Clifton [8] are appropriate for this and are modified to incorporate the characteristics of the BRB. There is also a slight difference in the tensile and compressive capacities of the brace, however these differences are minor and much less effect on the design of V braced configurations compared with conventional CBF systems. Lastly, the ability to specify the strength of the system to as close as the design force as possible will reduce the inherent overstrength of the overall system which has a benefit on the secondary members of the seismic resisting system and the foundations.

## Design and Testing Requirement

### United States Requirement

Currently, BRB system design in the United States is governed by the 2003 NEHRP Recommended Provisions for New Buildings and Other Structures (FEMA 450) and the 2005 AISC Seismic Provisions for Structural Steel Buildings. Furthermore, the Structural Engineers Association of Northern California (SEAONC) has established a document [9] on recommended provisions for BRB which contains the testing requirement for a BRB system to be acceptable for use. The loading sequence contained in this document, reproduced below, is used to determine whether the performance of their BRB system is adequate to fulfil the acceptance criteria set by AISC:

1. 6 cycles of loading at the deformation corresponding to  $D_b = D_{by}$
2. 4 cycles of loading at the deformation corresponding to  $D_b = 0.50 D_{bm}$
3. 4 cycles of loading at the deformation corresponding to  $D_b = 1 D_{bm}$
4. 2 cycles of loading at the deformation corresponding to  $D_b = 1.5 D_{bm}$
5. Additional complete cycles of loading at the deformation corresponding to  $D_b = 1 D_{bm}$  as required for the Brace Test Specimen to achieve a cumulative inelastic axial deformation of at least 140 times the yield deformation (not required for the Sub-assembly Test Specimen)

The variables in the criteria are defined by AISC as follows:

$D_b$ : Deformation quantity used to control loading of the Test Specimen (total brace end rotation for the Sub-assembly Test Specimen; total brace axial deformation for the Brace Test Specimen)

$D_{bm}$ : Value of deformation quantity,  $D_b$ , corresponding to the Design Storey Drift

$D_{by}$ : Value of deformation quantity,  $D_b$ , at first significant yield of Test Specimen

A comprehensive design guide on seismic design of the BRB system has also been developed by Lopez and Sabelli [7] in conjunction with the 2005 AISC Seismic Provisions for Structural Steel Buildings. In addition, Choi and Kim [10] proposed a BRB design method based on energy requirements, while Maley *et al.* [11] developed a displacement-based design method for dual systems with BRB and moment-resisting frames.

It should be noted that the current AISC Standard is more stringent than the provision used in this research project and requires the specimen to be tested up to  $2.0 D_{bm}$ . Given the observed performance of both specimens the authors would expect both would have achieved this increased deformation without a change in the observed behaviour.

### New Zealand Requirement

To the best of the authors' knowledge, there are no experimentally validated requirements or appropriate guides on designing BRBs in New Zealand, which was the principal motivation for undertaking this project. Several papers have been published in New Zealand regarding BRB systems based on overseas research and design procedures. Fussell [12] provides an overview of the BRB system, reviewing its concept, development and seismic performance, but more importantly, he also provides a short design guide for the BRB system. This design guide, however, does not prove its reliability through experimental testing or analytical modelling and does not include design examples.

## Connection Configuration

Xie [6] found that there are three common configurations for BRB end connections. The advantages and disadvantages of each type of connection are listed in Table 1.

### CASE STUDY BUILDING

A case study of a multi-storey building has been used to develop the design procedure in this research project. The 10 storey building is 36 m high (Figure 4) and is symmetrical about both principal plan axes. Along each axis, an inverted V-braced BRB structural system is utilised for resistance to lateral forces and thus each frame will resist half of the seismic actions. The structure is composed entirely of steel frames with composite floor slabs. The design of the composite floor slabs and gravity supporting columns is not considered in this research project.

The Christchurch earthquake series has provided a comprehensive set of data that can be used in time history analyses of the designed structure, despite the work being outside the scope of this project. Hence, the case study building is located at Christchurch with a soft or deep (Class D) soil type and hazard factor (Z) of 0.22 in accordance with NZS1170.5 (i.e. before the increasing of the Z factor for Christchurch city). The case study structure is an office-type building with an importance level of 2 according to NZS 1170.5 and is designed for a working life of 50 years.

Table 1: Different types of BRB connection [6].

Type of Connection	Advantages	Disadvantages
Standard Bolted	Bigger holes allow more erection tolerance compared to pinned connections	As it is not a true pinned connection, secondary moment forms between the connection and brace
	Multiple bolts provides more redundancy	High installation cost due to the number of bolts and splices
	Better distribution of forces to the gusset plate	Larger gusset plates to accommodate the bolts, and hence shorter core length, resulting in larger strains compared to pinned connections
Modified Bolted	Same as standard bolted No splices and significantly fewer bolts	Same as standard bolted
True Pin	Longer BRB core length resulting in smaller strains for a given load, eliminates overturning moment and less installation cost	Smaller erection tolerance

Time history analyses of different configurations performed by Deulkar *et al.* [13] found that the inverted V-braced type BRB configuration produces the least amount of roof displacement as compared to other types of configurations. Thus, the configuration has been chosen to provide good control in lateral displacement of the structure. It also suits the bay dimension (width and height) and is a common layout for CBF and EBF buildings.

The proposed BRB design procedure is based on the CBF design procedure contained in HERA Report R4-76 [8, with modifications to account for BRB design requirements.

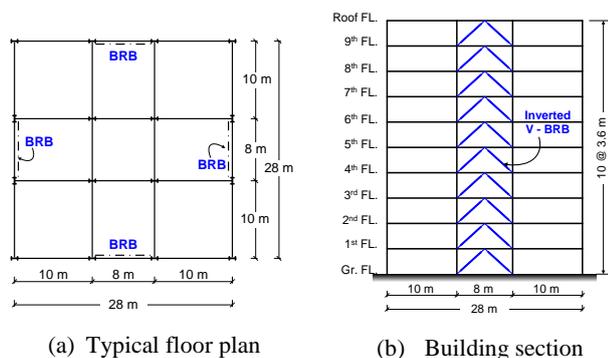


Figure 4: Floor plan and section of the building.

The static design force for all structural members is determined by a static equilibrium approach, involving either the equivalent static method or the modal response spectrum method. For this project, the equivalent static force based design method of NZS1170.5 Clause 6.2 was used.

The BRB design requirements contained in this paper also consider all the recommendations by Fussell [12] on the design of BRBs.

### DESIGN PROCEDURE

The design procedure is composed of two parts. The first part deals with the preliminary design of the structure, while the second part focuses on the design of the structural members.

The BRB system can be considered as either a fully ductile system or a system with limited ductility. Thus, the system is classified as a category 1 or category 2 structure according to the draft of NZS3404.7, as shown in Table 2.

Table 2: Relationship between structure category and member category for BRB systems (NZS3404.7, 2011).

Category of Structure	Category of Member		
	Braces	Collector Beams	Columns
1	1	2	2
2	2	3	2

The design procedure will closely follow the methodology to design category 1 and category 2 V-braced CBFs with braces effective in compression and tension contained in HERA report R4-76 Section 17.

### Finding the Height Limitation

Due to the BRB ability to suppress buckling it can be considered as a system with a compression brace slenderness ratio  $\leq 30$  when implementing the NZS 3404 provisions. Furthermore, because the inelastic strength and stiffness in tension and compression are identical, the height limits for category 3 response apply to a category 1 or 2 system. Therefore, the height limitations of Table 12.12.4.(3) of NZS3404 apply and allow a V-braced system of up to 12 storeys. The design example building complies with these recommendations.

### Determination of the $C_s$ Factor

The factor  $C_s$  found in C12.12.3 of NZS3404 accounts for inelastic behaviour of a CBF system, recognising that this behaviour is less stable and dependable than for an eccentrically braced frame or moment resisting frame system. In the design of BRB, due to the system ability to suppress buckling and using capacity design to design collector beams and columns for suppression of inelastic demand, this value can be taken as 1, thus being equivalent to that for EBFs.

### Preliminary Lateral Force

Initially, a time period is calculated using Equation 3.4 from NZS1170.5 Commentary Clause C4.1.2.2, which as expected was found to underestimate the fundamental period of the system.

$$T = 1.25k_t(h_n)^4 \quad (1)$$

where  $T$  = translational period of vibration (s);

$k_t = 0.05$  for a CBF; and

$h_n$  = height from the base of the structure to the top of structure, level  $n$  (m).

A structural system design ductility factor of 3 has been chosen for the design example, based on the recommendations of Clifton [14] to develop low damage designs. This is less than the maximum structural ductility factor of 4 recommended by Fussell [12]. The design procedure also incorporates design actions determined from structural ductility factors of 2.4 and 2 for the preliminary sizing of beams and columns, respectively. The lesser values of ductility are chosen to anticipate the increase in size of the member due to the over strength factor required for the capacity design procedure, which is used to determine the final sizes of the beams and columns. The empirical period is used for preliminary design, after which the period is determined from elastic analysis or Rayleigh method for final design and changes made to the design seismic loading if required.

After the design and the maximum horizontal seismic loads are calculated for a particular frame, the various beams, braces and columns can be sized. The methods for doing this are given later in this section. For conventional steel framed seismic resisting systems, this involves determining over which levels the primary seismic resisting system element (eg the active link in an eccentrically braced frame) will be made the same size, averaging the design seismic actions over those levels then matching the design capacity of the section chosen as closely as practicable to the averaged design seismic action [14]. However, for BRB systems the design capacity of the primary element (the yielding core) can be matched closely to the design seismic actions from analysis on each level, thus making the averaging unnecessary. In this project, averaging was applied in the initial building design, with the sections split into 3-3-4, going from top to bottom, meaning that the top three storeys have the same design force, which had been averaged from the initial design force from the top three storeys. The next three storeys and lower four storeys are designed by considering the average design force on the corresponding storeys, respectively.

### Structural Analysis

A structural analysis is performed in this step to assess the P-Delta Effects and to check the seismic lateral deflections. The program used for this analysis is SAP 2000.

The BRB elements can be separated into the yielding and non-yielding elements. When subjected to inelastic demand, the inelastic deformation on the BRB elements is principally limited to the yielding region of the steel core. Therefore, it is important to model the BRB as yielding length and not work-to-work point length as shown in Figure 5. A ratio in the order of 0.5~0.7 for the range of yielding length to work point length and a ratio of 2.0~1.4 corresponding to the effective stiffness of a braced element with the length ratio are recommended (Fussell [12]).

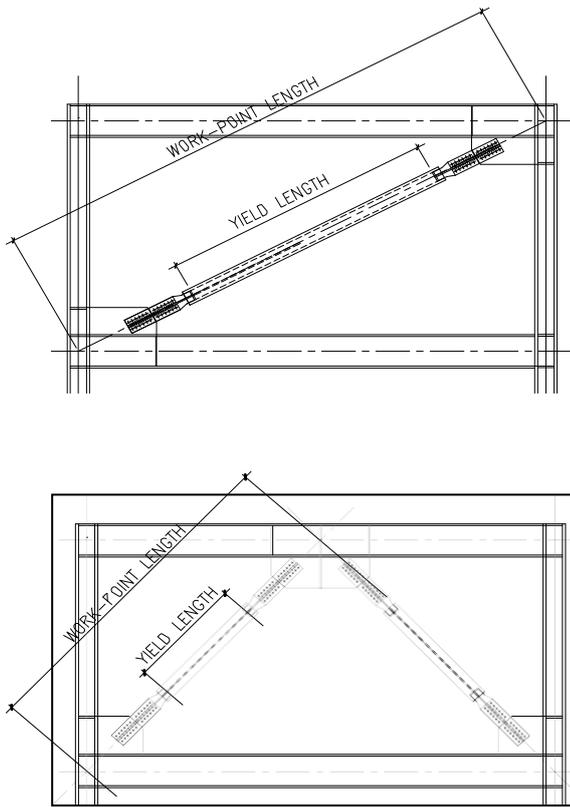


Figure 5: Difference between yield length and work-to-work point length [12].

This is quite a range in recommended stiffness parameters and, to determine the most appropriate values to use in modelling the BRB elements, several variations on these parameters in the case study building have been utilised. Two different ways of modelling the BRB elements were used, where the BRB elements have been constructed as a continuous member with the area of the steel core multiplied by 1.5. The second model is segmented, where each BRB element has been split into three sections with a length ratio of 0.2:0.6:0.2 for non-yielding; yielding; non-yielding components. All models were given realistic offsets to locate the ends of the brace elements. The simply supported and propped cantilevers are design options for the collector beams that can be chosen and their significance will be discussed later in this section. The steel core cross-section used in the analyses is of a cruciform shape before it was changed to a rectangle in the detailing. Collector beams can be chosen as either not supported in the midspan by the braces for gravity loading or supported by the braces and the significance of this is covered in the Collector Beam Design Actions section.

Each gravity outer (corner) column of the model has been modelled as a compound column including the contribution from the stiffness of the other columns supporting gravity load over half of the floor plan area, in order to include the strength and stiffness of these columns in the 2D model.

**Brace Member Design**

The steel core area of the brace shown in section C-C (Figure 6) is rectangular in these designs. The capacity of the steel core area can be easily determined by the formula shown in Equation 2 and requires the compression and tension capacity to resist the entire axial force running through the brace. Due to the ability of the BRB to suppress buckling, the tension and compression capacity of the steel core will be equal. A strength reduction factor of 0.9 is required by NZS3404.

$$N^* < \phi N_t = \phi N_c = \phi f_y A_{sc} \tag{2}$$

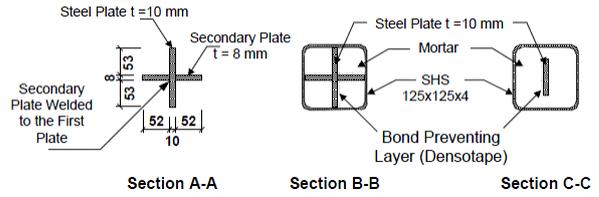


Figure 6: Different sections of a BRB element.

where  $\phi$  = strength reduction factor  
 $N_c$  = nominal member capacity in compression;  
 $N_t$  = nominal section capacity in tension; and  
 $A_{sc}$  = cross-section area of BRB steel core

In the same figure, the non-yielding elements shown throughout section A-A and B-B are designed by the same equation, but must resist the design action from the core after the overstrength factor is applied.

The design of the steel tube which surrounds the mortar and the steel core is in accordance with [12] on the design of BRBs. This method determines the stiffness required to be possessed by the casing to prevent the steel core from buckling. This check must be made for both in-plane and out-of-plane buckling if the casing is not doubly symmetrical. The formula shown in Equation 3 is derived from the relationship between the Euler buckling load of the casing and the nominal section capacity of the brace. This equation calculates the minimum moment of inertia required for the casing.

$$I_{min} = \frac{1.5L^2 f_y A_{sc}}{\pi^2 E} \tag{3}$$

where  $L$  = member length for assessment of slenderness in compression, in this case distance from centroid of top and bottom beam column joint as shown in Figure 5 as work-point-length; and  
 $A_{sc}$  = cross-section area of BRB steel core

Note that, following the testing of the two connection systems, it is recommended that the  $I_{min}$  for a pinned brace is increased by 1.25 times the value from Equation 3. For further details, see Section 5.2.

The bending capacity of the steel tube must also be able to resist bending forces generated by a transverse load equivalent to 2.5% of the brace axial force applied as a point load in the middle of the tube (Equation 4) in accordance with the compression restraint provisions of NZS3404 Clause 6.7.

$$\phi M_{casing} > 0.025 \phi_{oms} f_y A_s \tag{4}$$

where  $M_{casing}$  = nominal moment capacity of the BRB casing; and  
 $\phi_{oms}$  = overstrength factor from NZS 3404 incorporating statistical variation in yield stress and an allowance for strength increase due to strain hardening.

For the case study building the steel tube sizes has been chosen from the list of readily available sizes of square hollow sections (SHS) on New Zealand. However, the case study found that the size of the steel tubes most likely depends on the size of non-yielding elements due to detailing requirements.

**Capacity Design**

This step implements the capacity design procedures. In this case study, the yielding element of the BRB has been specified

as the weakest link in the structural system. Following the Capacity Design concept, the yielding element must behave in a ductile manner and the strength of the structure depends on the steel core area size. All other structural members are designed for the Capacity Design derived design actions from the steel core in accordance with NZS 3404 and HERA Report R4-76 [8] to ensure that the chosen ductile failure mechanism develops. Allowance should also be made to account for greater strength of the material than specified. Therefore, the strength of other structural members in the system is required to be greater than the maximum action that can be resisted by the steel core, taking into account a higher specified strength of material and increase in strength due to strain hardening.

The overstrength ( $\phi_{oms}$ ) used to determine the size of non-yielding elements is 1.25. This value corresponds to a category 1 member and steel grade of 300 MPa from Australia and New Zealand and is taken from Table 12.2.8(1) of NZS3404.

### Collector Beam Design Actions

Upon completion of the analysis, it was found that designing the collector beam for the axial loads from analysis using  $\mu = 2.4$  to find the preliminary design axial forces generates a slightly understrength beam when it is treated as simply supported and hence the gravity loading does not contribute to additional design axial forces to the brace. However, designing the beam as a propped cantilever and accounting for the axial load from gravity loading caused the axial load design to be severely underestimated when compared to the initial axial force found when  $\mu = 2.4$ . This is due to the axial force of the collector beams being dependent on the overstrength axial forces of the braces, allowing the propped cantilever design option to have a stronger brace as compared to the simply supported option. Further details on these two options and comparison of the preliminary sizes of the beams found using the suggested approach and the final sizes of the collector beam found from the capacity design procedures can be found in the first author's dissertation [15].

This design procedure strongly recommends the use of the simply supported design option. Considering the beam as simply supported throughout the whole length is the most conservative way to estimate the design bending moments from gravity loading. Despite providing a larger beam, the simply supported design option will provide a smaller brace which reduces the overstrength design action for the column and connection of the system. Stronger and stiffer collector beams, in conjunction with semi-rigid bolted connections to the columns, also are expected to contribute to the self-centring ability of the overall system, although this has not been specifically investigated in the research project. The larger beam is less likely to deform under severe loading and provides a stronger anchor for the braces to return to its original position compared to the smaller beam. The simply supported design option for the collector beams under gravity loading is thus expected to perform better in earthquakes. Time history analyses will be required to confirm this proposition.

### Collector Beam Capacity Design

The case study design uses the method suggested by HERA report R4-76 [8] with the post buckling compression factor,  $\alpha_c = 1.0$  to find the critical design shear. BRB design with high drift should also consider the shear force generated by the plastic hinge formed in the collector beam as per Fussell's recommendation [12]. Usually, the additional collector beam shear force can be ignored as plastic hinges should not form in BRB systems, however these hinges may occur where there are significant rotation on the connection of the braces due to large lateral displacement of the frame. It is unlikely that the increased collector beam shear force influences the design of the collector beam, however this additional force will

significantly increase the column design actions as the structure height increases.

### Column Design Actions

This design procedure recommends that the column design bending moments are found using the method recommended by the HERA report R4-76, where the eccentricity of the applied resultant axial force from the collector beam and brace is considered along with collector beam shear forces to the column centrelines. Note that in the design example this procedure has not been followed and the design bending moments are obtained from the SAP2000 analyses, as the design moments are usually very small.

### Column Design Capacity Design

The columns will be designed in accordance to the relevant sections of NZS3404 as stated in the R4-76. The design of the columns is most likely governed by the combined actions of axial forces and bending moments. The other requirements stated in the HERA report R4-76 design guide should also be satisfied where applicable and other load combinations considered. The post buckled compression capacity factor  $\alpha_c = 1.0$  is used.

### Maintaining a Strong Column System

It is important to maintain a strong column in the frame supporting the BRB. As shown in Figure 8, it is likely that the design moment capacity of the collector beam will be greater than the design moment capacity of the column at a given level. If the collector beam to column connection is a welded moment connection, under high inelastic rotation the column will develop a plastic hinge in preference to the collector beam. This increases the likelihood of a soft storey forming if anything happens to the BRB which is undesirable. To avoid this either a semi-rigid connection between the collector beam and column should be used with the column designed for the overstrength from the connection or the moment capacity of the collector beam be selectively weakened near the welded moment connection to the column (eg through a dog bone type reduction in beam flange width) so that the strong column hierarchy is maintained.

## DETAILING OF BRB

Key recommendations for detailing and construction of the BRB system have been included and detailed in Wijanto [16], including construction methodology and material specification for de-bonding agent and encasing mortar. In addition, if the BRB frame system is located in external environments, it is recommended to apply coating to the system to prevent corrosion as recommended by section 12.15 of HERA Report R4-133 [16]. This paper will only provide a brief overview on the detailing of pinned and bolted specimen shown on Figure 7 and 8, respectively. Eccentricity introduced through the connections has been included in the model.

### Detailing to prevent lateral buckling

The complete brace was designed to prevent buckling as a pinned column buckling between effective points of restraint at the end connections, i.e. in first mode Euler buckled shape, as shown in the lower diagram of Figure 9. However, when loaded, the pinned specimen buckled at the clamping plates in a sidesway failure as shown in the upper diagram in Figure 9.

This mode of failure is unacceptable as the failure of the entire system now depends on the strength of the clamping plates to resist local buckling, which is weaker than the ability of the encased steel core to resist lateral buckling. To prevent this undesirable mode of failure, the non-yielding segment reinforcement lengths were extended on each side of both ends of the pinned specimen. This was achieved by welding an

extra rectangular plate to provide continuous lateral restraint, up to 5 mm from the pins, shown in Figure 7. This suppressed further buckling of these plates and is part of the recommendations for a pinned connection.

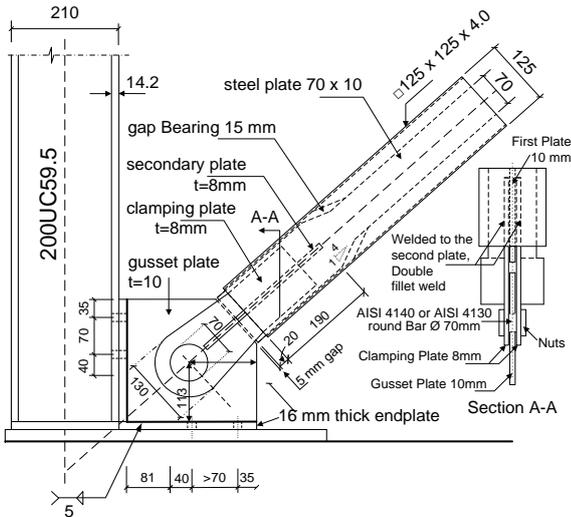


Figure 7: Pinned specimen connection details.

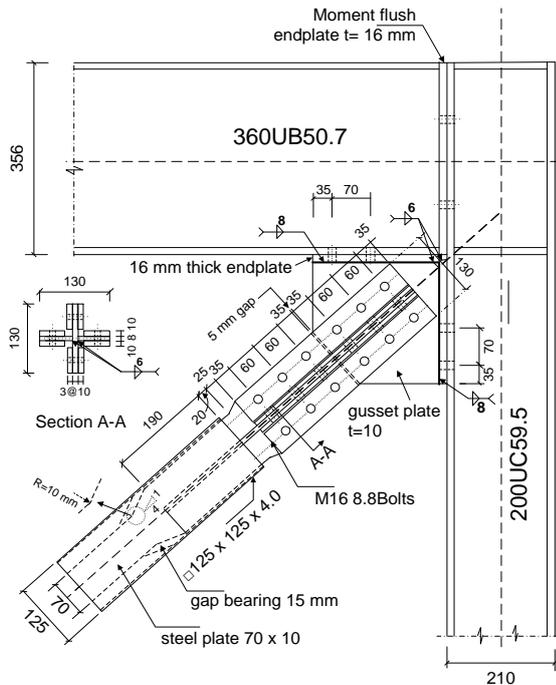


Figure 8: Bolted specimen connection details.

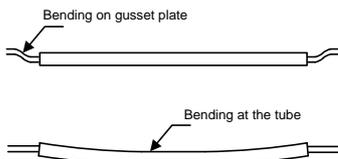


Figure 9: Modes of failure.

For the bolted specimen, the cruciform shape of the non-yielding segment was successful in preventing bending on the gusset plate. This was due to the wings of the non-yielding segment providing additional restraint to prevent the gusset plate from buckling when the specimen was loaded axially.

**Prevention of Slackness in the Connections under Inelastic Cyclic Loading**

The test results discussed in the next section show that without attention to connection details the connections will develop slack under repeated cyclic loading leading to undesirable flexibility of the overall system. Therefore, several changes are made to the proposed procedure post sub-assembly testing to prevent slack from happening.

The pin was designed with a factor for pin rotation ( $k_p$ ) of 1.0, which does not allow for rotational motion. It is recommended in the future to take  $k_p$  as 0.5, which will increase the size of the pin and ensure that the pin can rotate under loading. This will reduce the chance of slack in the system developing under increasing cyclic loading due to slight elongation of the pin hole in the plys, which was the observed source of this slack. The pin itself did not suffer any visible distortion or surface damage

The 4-M16 Grade 8.8 bolts which are used to connect the beam and column were unable to be fully tensioned (which is a design requirement) due to inadequate space in the connection, preventing the use of mechanical tools to fully tension the bolts as required. Future designs must take account of this to provide sufficient space between the bolts and the surrounding structural members, to allow the insertion of bolt-tensioning equipment. Fully tensioned bolts designed in tension bearing for the over-strength generated by the brace core and detailed as slip-critical connections will effectively eliminate the slack in the system, as observed in all bolted fully tensioned bolted splices from the Christchurch earthquake series [1].

The moment endplates and gusset plates were previously designed for over-strength of the brace force and to only suppress the Mode 3 (bolt failure), while still allowing Mode 1 (complete flange yielding) and Mode 2 (bolt failure with flange yielding) to be the critical mode of failure, as reported in literature [17, 18]. In the future, the procedure should be based on suppressing both the Mode 2 and Mode 3 failures for the endplates on the beams and suppressing mode 3 behaviour for the flanges in the supporting columns. This will limit the failure to endplate yielding and no bolt extension, which will prevent slack. The bolts will not undergo extension, and the endplates will remain elastic to a frame rotation of approximately 15 milliradians, which is expected to assist in the self-centring of the brace and ensure that the beam and column does not deform.

The pinned specimen had been shown to exhibit an increased lateral displacement at the middle of the tube compared to the bolted specimen as shown in Figure 5.31 and 5.32 in [15]. The elastic stiffness limit for the braces used in their design (see Brace Member Design) should be increased by 1.25 for the pinned brace for both the in-plane and out-of-plane directions, to account for the lower elastic rotational support from the end connections and supporting frame, and the different in-plane modes of elastic buckling.

**SUB-ASSEMBLAGE TESTING**

The sub-assembly specimens are designed to resist lateral forces induced by earthquakes and to behave and perform like a generic BRB system. In the experimental test, the earthquake forces were simulated using a Shore Western Short Ended 923.5-22.58 dynamic actuator. Only two specimens with different connection types have been constructed for the experimental test due to the cost and time required, and hence these two specimens were used for the entire experimental test with no replacement. This was less than ideal as each subsequent test will cumulatively bear some effects from earlier tests; however, it was a good opportunity to examine the ability of the BRBs to withstand multiple severe events

separated in time, which will be required if the braces are to remain in place following a severe earthquake. The column and beam have also been re-used in all the testing.

### Test Setup

A series of instruments were used to monitor and record the performance of the sub-assembly specimen. Two Linear Variable Differential Transformers (LVDT) were used to measure the lateral deflection of the sub-assembly frame. As shown in Figure 10 and 11, one is built in the actuator and the other was installed at the top of the right column. The actuator included a built-in load cell to measure the forces applied to the frame.

Two displacement measuring device were used to measure the in-plane and lateral buckling at the centre of the specimen, and portal gauges were used to determine whether the baseplates moved during the experimental test. The test rig was supported on a customised baseplate, with holes which line up with stress bar holes on the strong floor.

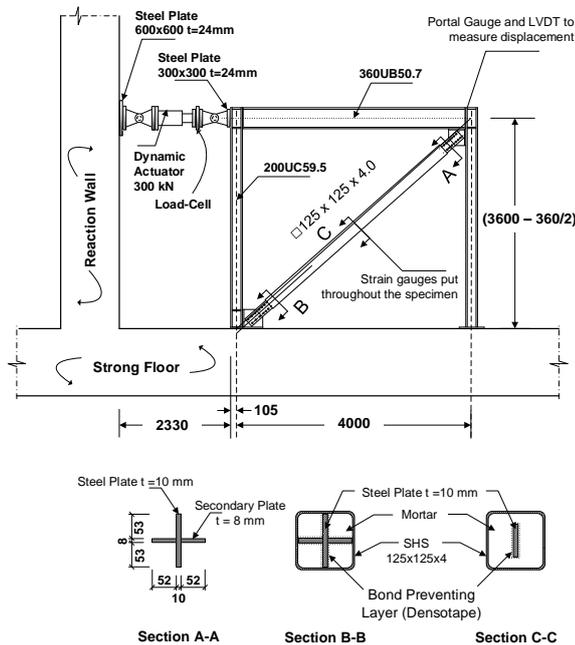


Figure 10: Elevation plan of the test set up.

### Loading Regime

Each specimen was subjected to three different loading regimes: trial static, cyclic static (at pseudo-static rates of loading, being  $1/50^{\text{th}}$  of the dynamic rate) and cyclic dynamic (at earthquake rates of loading, typically 1 second per cycle in the inelastic range) as shown in Figure 12 and 13.

The loading regime was implemented initially on static tests and then finally on testing at seismic dynamic rates of loading. In the static testing, the frame was loaded slowly to reach the 54 mm displacement in each direction, corresponding to 1.5  $D_{\text{bm}}$  and is the largest displacement in loading regime as stated by SEAONC [9]. In this case,  $D_{\text{bm}}$  is taken as 36 mm, which corresponds to 0.01 times the storey height. A loading rate of approximately 2 mm per minute has been used for both specimens.

The magnitudes of the loading regimes are based on the document released by SEAONC [9] on the testing of BRB systems. The specified loading was applied by the dynamic actuator shown in Figure 10, which has a maximum capacity of 250 kN (rated at 300 kN, but the pump size is too small to

allow the rated capacity to be developed), and is controlled by displacement. The test series was performed over three weeks, with the exception of the strain ageing test. In chronological order, the first test conducted was the trial test of the pinned specimen, followed by both bare frame tests. A second static trial test was performed on the pinned specimen, and the cyclic static test of the pinned specimen followed after. The bolted specimen was then installed and tested for all of the loading regimes. Lastly, the pinned specimen was dynamically tested and left for two and a half weeks in an average and near constant temperature of 22°C, before it was tested with the same dynamic loading regime to investigate the effect of partial strain ageing. (Ideally it should have been left for three months at that temperature, but there was insufficient time for that within the project timeframe.)



Figure 11: Sub-assembly test model at the laboratory.

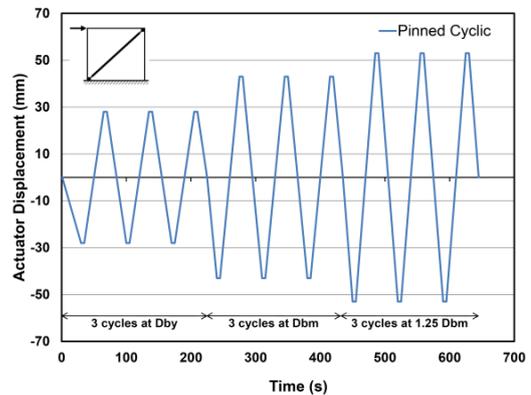


Figure 12: Cyclic static test loading regime.

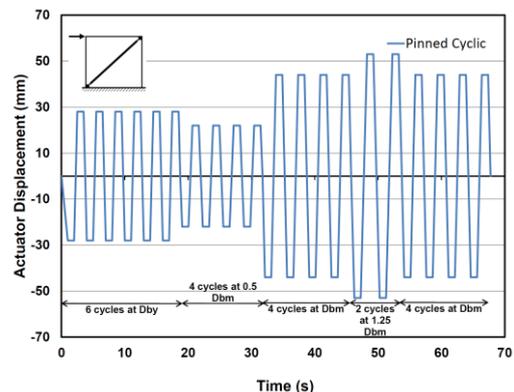
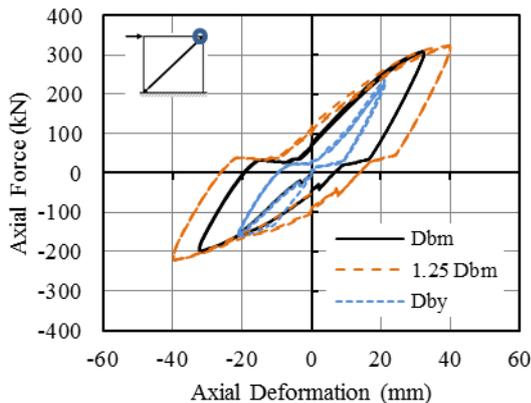


Figure 13: Cyclic dynamic loading regime.

### CYCLIC STATIC SUB-ASSEMBLAGE TESTING

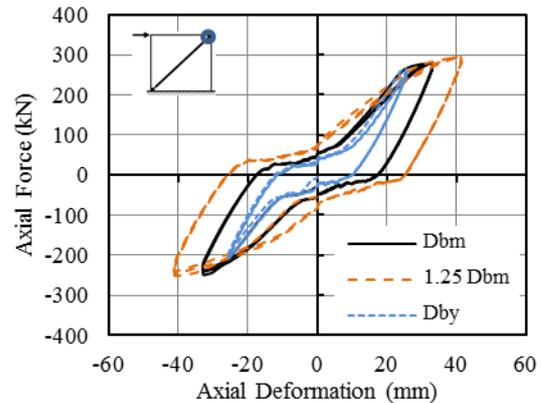
Each brace specimen exhibited a stable and repeatable hysteresis loop as shown in Figure 14 and 15, which is characteristic of the BRB system. The steel core on each specimen also had not failed or buckled in compression, despite the high forces subjected onto them, which shows that the proposed restraining mechanism is functional. The sliding of the steel plates in and out of the steel tube and encasing mortar also shows that the de-bonding materials performed well. The most interesting result from the cyclic test was the observation that both specimens were stronger in tension than in compression; the reverse effect to that generally reported in the literature [5]. Due to the absence of strain gauges and limited measuring channel to record the brace extension, the reasons for this difference in behaviour could not be absolutely determined, although three explanations are given. Firstly, the steel tube may have contributed to the resistance more effectively in tension than in compression; due to the development of end moments resulting from frame action. Secondly the brace in tension puts the collector beam into compression which will have slightly increased the frame action from the gravity system compared with when the brace is in compression and the collector beam in tension. Few tests have been conducted on BRBs in complete frames tests on BRBs alone would not incorporate the frame effect. Finally, the steel core had a very high slenderness ratio in comparison to many other BRB tests, generating higher elastic buckling of the brace which may have reduced the resisted compression force. The testing of this very slender brace was deliberate; as there was only the opportunity to test one brace of each end connection type, the braces were selected to be as close to the slenderness limit as possible to determine the adequacy of the intended provisions to control compression buckling of the complete brace between its end restraints. More research and testing will be needed to determine the exact cause of this behaviours and whether the modifications to the proposed design procedures will be able to reduce the difference between the peak tension and compression forces.



**Figure 14:** *Hysteresis of pinned specimen for cyclic static test.*

Despite the hysteresis loops being stable and repeatable, unwanted slack was exhibited every time the specimen returned to its original position during the testing phase in both specimens, which reduces the system's ability to dissipate energy. The slack originates from several sources. The most obvious indication was the moment endplate and gusset plate to the column, which was designed to allow the partial bolt extension as its critical mode of failure (that is, mode 2 in accordance with HERA Report R4-133 [16]). When the braced frame was returned to its original position, the moment endplate and gusset plate to column snapped back, which caused a loud noise, and slack exhibited in the

hysteresis as a result. This was further supported by the damaged bolts found on the moment endplate connection after the test. The failure of the bolts may be caused due to the bolts barely passing the design check for combined action of shear and tension. The other source of slack was from the connection in each specimen. The bolts were not fully tensioned due to lack of access, but only hand tightened, while the pins were not designed for rotation which leads to more flexibility in the system for slack to happen due to smaller pin sizes. The design recommendation is to suppress both the Mode 2 and Mode 3 failures along with fully tensioning the bolts.



**Figure 15:** *Hysteresis of bolted specimen for cyclic static test.*

The cyclic tests revealed the buckling mode for the pinned and bolted specimens. Note that all buckling modes were mostly elastic as the specimens were still relatively straight at the end of the tests. The pin-ended specimen acts like a true pin, and exhibits the first mode of buckling in-plane, while the bolted connection specimen undergoes elastic double curvature buckling in-plane, with higher in-plane stiffness and, subsequently, lower displacements. In addition, the stiffer connection and the connecting plates in the bolted connection also lead to less in-plane movement for the bolted specimen as compared to the pinned specimen. The pinned specimen may also have undergone slight inelastic buckling in the duration of the test, as its in-plane movement did not oscillate in its original position.

The pinned specimens also exhibited a relatively large lateral movement when compared to the bolted specimen. In the out-of-plane direction, despite the behaviour of the bolted specimen being considered as a pinned connection, the bolted connections provided more out of plane rotational stiffness than the pinned specimen connections. In addition, the steel core length of the pinned specimen was longer compared to bolted specimen, which meant that the pinned specimen had a longer effective length and higher tendency to buckle than the bolted specimen.

The higher force required to pull the bolted specimen compared to the pinned specimen is due to the near equal and opposite end moments generated on the brace by frame action. These moments transfer some of the axial force from the core to the brace, thereby increasing the effective axial load carrying capacity of the brace. This occurs to a greater extent than in the pin ended brace. However, there was not a large difference in terms of the capacity for the bolted and pinned specimens. Despite this, from the lateral and in-plane movement of the brace, the bolted specimen which exhibited less movement was a more stable configuration. Therefore, it is considered that the bolted specimen performed better than the pinned specimen in the cyclic static test and is the recommended end connection detail.

### CYCLIC DYNAMIC AND PARTIAL STRAIN AGEING SUB-ASSEMBLAGE TESTING

Comparing Figure 14 and 15 with Figure 16 and 17, both specimens show hysteresis results which are very similar compared to that obtained from the cyclic static tests. The comparison between the bolted and pinned specimen performances are also consistent with the conclusion obtained from the cyclic static tests. The only difference was in terms of strength, where the sub-assembly frame loaded dynamically was stronger compared to a statically loaded frame for a given displacement. The most likely reason is that the frame in the dynamic test had already undergone strain hardening, after it had been subjected to the demand from the previous static loading regime. The increase in strength from dynamic loading at seismic-dynamic strain rates is typically below 5% [19].

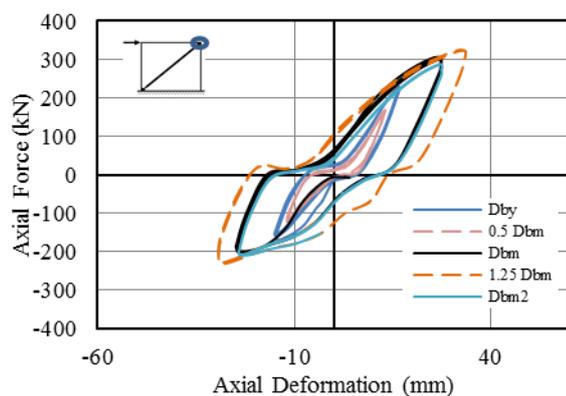


Figure 16: Hysteresis of pinned specimen for cyclic dynamic tests.

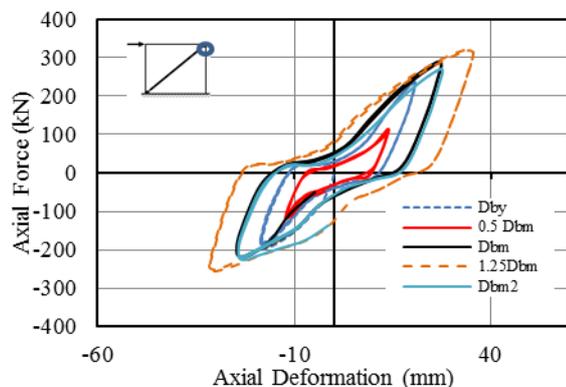


Figure 17: Hysteresis of bolted specimen for cyclic dynamic tests.

Due to the overestimation of the actuator strength, this research project had not been able to fulfil the SEAONC loading requirement. It has only been able to load the sub-assembly frame past the elastic and yielding region, and into the strain hardening region, but had not been able to load the brace to its ultimate stress and fracture. However, this was sufficient to confirm the reliability of the design procedure and detailing, as the steel core did not buckle, despite it being loaded with a force much higher than its compressive capacity if it had been resisting the load alone. Table 3 is a summary of the maximum force experienced by the brace specimen, accounting for the force reduction for the bare frame for each loading regime. The maximum compressive force of the pinned specimen was smaller than expected, despite it reaching the specified displacement, which may be attributed to the fact that it underwent a significant lateral movement.

Table 3: Maximum forces subjected to the specimen in different loading regimes

Loading Type	Pinned Specimen		Bolted Specimen	
	Tension (kN)	Comp. (kN)	Tension (kN)	Comp. (kN)
Static	282.44	231.3	234.9*	215.2*
Cyclic Static	263.8	207.4	291.8	251.8
Cyclic Dynamic	324.9	227.8	320.4	256.0

Note: Yielding force of the steel core is 245 kN and the static bolted specimen test had to be stopped prematurely due to premature failure of the actuator.

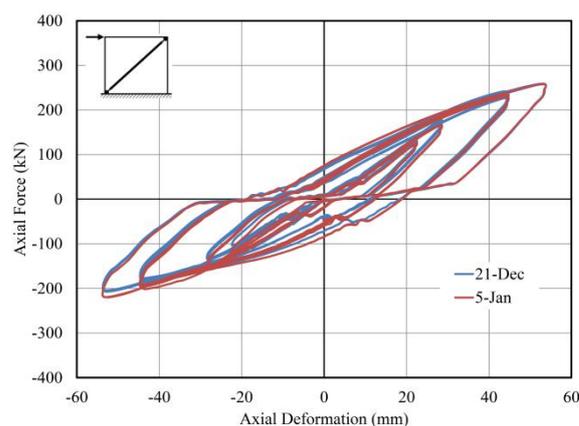


Figure 18: Test results of pinned specimen with partial strain ageing.

From Figure 18, the effect of partial strain ageing is shown. The 05/01/2012 test revealed that the pinned specimen was only very slightly stronger compared to the tests performed on 21/12/2011. The interval between tests of 15 days and the 22<sup>0</sup> C temperature of the steel would have allowed some 25% of the full strain ageing to develop and the influence of this is minimal.

### CONCLUSIONS

This project aims to commercialise BRB system in New Zealand through the development and confirmation of performance, by testing a generic design procedure for the system in compliance with New Zealand standards. The experimental test results shows that several improvements need to be made to the proposed design procedure and detailing as outlined throughout the paper; especially the procedural modification to prevent slack from occurring. However, the proposed generic design procedure has fulfilled its purpose, which is to allow New Zealand structural engineers to follow tested design procedures, with recommended improvements, to design the BRB system without relying completely on proprietary systems and in accordance with New Zealand Standards.

Several conclusions can also be drawn from the sub-assembly testing of the specimen, outlined below:

- The experimental test results show stable hysteresis loops, but with slightly different capacities in compression and tension, which may be caused by the brace slenderness ratio. Strain gauges are needed to confirm this hypothesis
- Due to the underperformance of the actuator, the brace specimens were not able to be tested to its full potential. However, the post mortem reveals that the brace core was undamaged and not visibly deformed, which meant that

the system is capable of taking a higher load than that tested in this project, if necessary.

- The system developed slack due to the flexibility in the connection, but procedural changes have been recommended to prevent this in a properly designed pinned or bolted connection contained in section 5.
- The load rate effect on the BRB strength is minimal
- The effect of partial strain ageing is minimal
- Bolted and pinned specimens display only minor difference in their ability to resist forces. However, despite the pinned specimen having an easier construction, installation and detailing process, currently the bolted specimen performs better in terms of stability, and thus will be recommended over the pinned specimen.
- Procedural changes may allow the pinned specimens to perform as well as the bolted specimens but the pinned systems still require closer attention to tolerance during construction and the probable use of shims to achieve the tolerances required.

### RECOMMENDATIONS FOR FUTURE TESTING AND RESEARCH

The next stage of research should focus on the following:

1. Numerical modelling of complete systems to investigate the overall response, the influence of different collector beam design options (simply supported or propped for gravity loading) and the self centering ability of the system
2. The development of light weight BRB units for use in retrofit
3. The development of dynamically self centering BRB units.

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